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Date: July 18, 2008

Subject: Geotechnical Engineering Alignment Report (464)
Lone Pine North and East
MT-STPP 36-1(11)26
UPN 1289

The enclosed geotechnical report provides our recommendations for completion of Activity 464 (Geotechnical - Alignment). This report and enclosed attachments describes our field investigations, subsurface conditions encountered, laboratory testing performed, and our recommendations for cut and fill slopes, embankment foundations, culvert foundations, subgrade treatments, and general recommendations for construction. Recommendations that include different options have also been provided in select locations. After review by those on the distribution list, we should be notified of any comments or preferences with respect to the options so that we can provide additional recommendations. We will provide appropriate special provisions after we have received any comments or concerns from the design team.

This geotechnical report is based upon the existing alignment and grade, in the event changes to the alignment or grade are required, the Geotechnical Section should be notified to review the changes, and determine if they affect the recommendations contained within this report.

A memorandum completed in 1999 was provided to the Bridge Bureau for design of the bridge foundations. This memorandum will be updated with revised recommendations and will be completed as part of our activity 466 work.

1.0. PROJECT LOCATION AND INFORMATION

1.1. Project Location. The project is located on MT Hwy 28 in Sanders, Lake and Flathead Counties. The project begins at about RP 26.0 just north of Lonepine and extends north to Niarada and then curves east about 2 miles to RP 35.8. The project is located within the Flathead Indian Reservation.

The project limits are Station 31+40.95 to Station 547+20.00 on STPS 36-1(4)26.

1.2. Proposed Scope of Work. The scope of the project is to reconstruct approximately 9.8 miles of MT Hwy 28. The road is severely deteriorated and will be upgraded to meet modern geometric standards. The work includes grading, gravel surfacing, plant mix surfacing, culverts, the replacement of one bridge over the Little Bitterroot River and miscellaneous other features. Original geotechnical recommendations for the bridge were presented in a memo from John Moran and Joe Olsen to Joseph Kolman, dated August 20, 1999. The analyses performed and recommendations provided will be revised and new recommendations will be provided for this bridge as part of our Geotechnical Activity 466 report.

The proposed alignment centerline will be offset from the Present Traveled Way (PTW) in many locations, but for most areas will utilize the PTW within the new embankment prism. The proposed alignment will contain significant changes to the existing horizontal and vertical alignments.

Earthwork will consist of fill placement to maximum heights on the order of 25 feet and excavations with maximum cut heights of approximately 30 feet. Standard cut and fill slopes appropriate for this route have for the most part been used. Exceptions to these standard slopes have been provided in the approved Design Exceptions report. Depending on the height of fill, the proposed fill slopes for this project are either 6:1, 4:1 or 3:1. Proposed cut slopes are 2:1 or flatter.

The proposed new bridge on the Project crosses the Little Bitterroot River at approximate Station 64+00 (R.P. 26.5). It will replace an existing timber bridge with a new single span concrete bridge which will be about 82 feet long. The new bridge will be set so that it has a low beam elevation of 2763 ± feet and it will be located within a vertical curve.

In addition to the bridge, drainage work will include construction of culverts. The Hydraulics Section has recommended the following culvert options:

123+00: 36-inch RCP or CSP Irr.
End Treatment - FETS
Pipe Skew 0 degrees

156+30: 48-inch RCP, CSP, or CAP Irr.
End Treatment - FETS
Pipe Skew 0 degrees

180+00: 48-inch RCP Irr.
End Treatment – FETS
Pipe Skew 0 degrees

367+14: 13’-6” x 9’-6” SSPPA
6” x 2” corrugations
Pipe Skew 25 degrees Rt.

The remaining mainline culverts will be the minimum size of 24” RCP, CSP, or CAP. Approach pipes will be 18” RCP, CSP, CAP, or HDPE.

Stock passes will also be constructed in several locations.

The MDT Pavement Analysis Section has recommended the following surfacing typical section:

0.30 ft - Plant Mix Bituminous Surfacing
0.85 ft - Crushed Aggregate Course
2.00 ft - Special Borrow (R $\geq$ 60, soil class A-1-a)
3.15 ft - Total

1.3. Geotechnical Summary. The soils along the alignment consist mostly of Quaternary Glacial Lake Missoula silt and clay deposits. The silt and clay deposits are compressible and are highly moisture sensitive. The silt soils are also highly susceptive to frost heave. Peat and very soft organic silt was encountered in a swampy area between stations 240+00 and 253+00. Our recommendations for design and construction in these soil conditions are discussed in the text of the report.

2.0 GEOTECHNICAL INVESTIGATION AND SUBSURFACE CONDITIONS

2.1. Geology and Site Conditions. The section of Hwy 28 from Lonepine north to Niarada traverses the Little Bitterroot River Valley west of the Salish Mountains. Northeast of Niarada, the highway follows the Big Draw, a pre-glacial valley of the Flathead River (Hyndman, 1986). The entire length of the project crosses over deep (100-300 feet thick) silt and clay sediments that settled out of the Glacial Lake Missoula Water column during the last ice age. The silt and clay deposits are exposed in the cuts along the highway. Erosion of the sediments has given the area badlands topography. The land adjacent to the highway is primarily used for grazing.

A rock outcropping is present on the south side of the PTW northeast of Niarada from Station 490+00 to 500+00. The rock is Precambrian belt rock mapped as the Revett Formation (Harrison, Griggs, and Wells, 1986). The Revett Formation is described as white medium-grained quartzite interbedded with siltite and argillite.

The existing roadway is generally in poor condition with numerous longitudinal and traverse cracks, alligator cracking as well as evidence of long-term settlement, lateral displacement, and frost heaving. Overlays and/or patching are present intermittently throughout the project length.

The site lies within the Intermountain Seismic Belt and is mapped as a zone of potentially significant seismic ground movement. Based on the fine grained nature of the soils, we anticipate that the risk of liquefaction or of lateral spreading is low in most areas. Earthquake induced settlement and slope movement is possible due to loss of strength during dynamic loading in the fine grained soils.

2.2. Subsurface Investigation. The MDT Field Investigation Unit completed 36 geotechnical borings along the alignment. Boring logs for 1289-S1 through 1289-S15, 1289-S15A, 1289-S18 and 1289-19 were completed in November and December of 2007 with a CS 2000 truck-mounted drill rig or a CME 850 ATV drill rig. Borings 1289-1 through 1289-16 were completed along the alignment with a CME 850 ATV drill in January of 2006. Borings 4-4-99 and 4-5-99 were completed at the proposed Little Bitterroot River bridge location in February of 1999 with a CME 550 ATV drill. All of the geotechnical boring logs are provided in the Appendix. The depths of the geotechnical borings varied from 20 to 100 feet below existing grade. Hollow-stem auger techniques were used to advance the borings. Sampling consisted of split-spoon Standard Penetration Tests (SPT) and Shelby tubes. In-situ vane shear testing was also conducted in some of the boreholes.

The MDT District Materials Lab completed a total of 27 soil survey borings along the alignment between 1988 and 2006. The soil survey borings extended from 3 to 12 feet below existing grade. The soil survey borings were completed using a truck-mounted drill rig equipped with solid-stem augers. A summary of the soil survey, Materials Lab Form 111, is included in the attachments.

In addition, a Geotechnical Section representative performed hand auger borings of the subgrade and obtained grab samples from exposed cuts at 16 locations along the project. A table summarizing the results of the hand augers and grab samples is included in the attachments.

The Geotechnical Section also requested Falling Weight Deflectometer (FWD) data from the Non Destructive Testing Unit of the Surfacing Section to evaluate this information with respect to existing subgrade conditions along the project. The data evaluated was collected during June of 1998, September of 1998 and June of 2004. The data collected during June is assumed to be representative of a time of year when moisture contents and water levels are high. The FWD test imparts a load pulse by dropping a large weight, transmitting the energy to the pavement through a circular load plate, which simulates the load produced by a rolling vehicle wheel. Geophones mounted radially from the center of the load plate measure the deformation of the pavement in response to the load. The average Resilient Modulus (M_r) of the subgrade to a depth of about 48 inches can be back-calculated if the thickness of the pavement and base is known. The Resilient Modulus gives an idea of a soil's potential response to loading by heavy equipment during construction, and loading by traffic once the road has been constructed. Although this information is obtained through the existing PTW, the data can be used to obtain a general idea of subsurface conditions in the general vicinity.

Subgrade Resilient Moduli, as estimated from the FWD data varied widely throughout the project. Values ranged from an approximate minimum of 2,500 psi to an approximate maximum of 22,500 psi. Many of the values are less than 9,000 psi and nearly all the values obtained during the June data collection times are below this value. Values below 9,000 psi are generally indicative of soft, fine grained soils where relatively low shear strengths should be expected. A value of 4,500 to 5,000 psi roughly equates to a CBR value of 3 or R value of approximately 5. Resilient moduli values are consistently low from approximate Station 67+00 to 236+00.

2.3. Subsurface Conditions. Predominately Glacial Lake Missoula silt and clay deposits were encountered during the subsurface investigation. Peat and other highly organic deposits (AASHTO A-8) were also encountered in some of the borings.

2.3.a. Glacial Lake Missoula Sediments. The predominant soils along the alignment are Lacustrine Glacial Lake Missoula deposits consisting of A-4 (silt), A-6 (lean clay), and A-7-6 (lean and fat clay). A-7-5 (elastic silt) was also encountered sporadically throughout the project. The soils are very soft to very stiff with SPT N-values between 0 and 32 blows/foot. The average N-values were in the range of 4 to 8 blows per foot, indicating the soils are generally medium stiff, but the relatively consistency of the soils is highly dependent upon location within the project.

2.3.b. Organic Soils. Peat and other organic soils were encountered in borings 1289-7, 1289-8, 1289-11, 1289-12, HA-10, HA-11, 1289-S11 and 1289-S12,. The organic soils had moisture contents ranging from 50 to 338 percent, with an average of 116 percent. Organic soils are highly prone to long term secondary consolidation settlement that can continue for years or even decades and also exhibit low shear strengths.

2.3.c. Groundwater Observations. Groundwater was encountered in some of the geotechnical borings. Table 1 summarizes groundwater observations. The observations were made on the date(s) the borings were drilled. Groundwater levels will fluctuate over time. Seasonal and annual changes in groundwater conditions should be anticipated, especially during spring thaw and following heavy precipitation.

Table 1. Groundwater Observations.

Location	Boring No.	Ground Surface Elevation (ft)	Water Level Observations (ft)		
			While Drilling	Upon Completion	Remarks
Sta 55 +00, 40 ft RT	1289-S19	2795	25.0	30.0	-
Sta 61+00, 30 ft LT	1289-S18	2766	29.0	29.0	-
Sta 63+53, 34 ft RT	4-4-99	2757.3	9.0	2.5	Water depth @ 0.1 ft 24 hrs after completion of boring
Sta 64+70, 29 ft RT	4-5-99	2758.9	11.4	12.0	-
Sta 64+94, 10’ RT	1289-S15A	2762	ND	13.0	-
Sta 65+10, 10’ RT	1289-S15	2762	ND	13.0	-
Sta 155+80, on CL	1289-5	2813	4.0	0.0	-
Sta 212+43, 45’ LT	1289-S12	2812	11.0	13.2	-
Sta 243+00, 10 ft LT	1289-8	2817	7.5	3.0	Water on ground surface on east side of highway at time of boring
Sta 246+00, 40’ LT	1289-S11	2812’	9.5	4.0	-
Sta 250+00, 10 ft LT	1289-7	2818	5.0	3.0	Water on ground surface on east side of highway at time of boring
Sta. 253+00, 30’ LT	1289-S10	2825	2.5	2.5	-

Sta 367+35, 65 ft RT	1289-11	2845	10.0	5.6	Stream level about 3 ft below top of boring elevation at time of boring
Sta 398+00, on CL	1289-12	2853	5.0	2.3	Water level on ground surface directly west of boring at time of boring
Sta 495+00, 75’ RT	1289-S2	2880	16.5	16.5	-
Sta 509+00, 5 ft LT	1289-15	2913	20.0	45.0	-
Sta 549+40, 60 ft RT	1289-16	2916	7.5	-	-

ND = Not detected
Notes:
1. Groundwater was not encountered in borings 1289-1, 1289-2, 1289-3, 1289-4, 1289-6, 1289-9, 1289-10,1289-13, 1289-14, 1289-S1, 1289-S3, 1289-S4, 1289-S6, 1289-S7 and 1289-S9 at the time of the investigation

The groundwater levels encountered during our drilling program are presented on the exploratory boring logs. These water levels are representative of the time and location where the boring was advanced. Groundwater levels will fluctuate in response to seasonal variations and may be encountered at different depths during construction. Water levels will likely be higher during spring conditions or in a year with higher precipitation.

The soils are especially poor in the area from approximate 240+00 to 253+00 and consist of peat underlain by organic silt. At many of the boring locations within this section, the SPT sampler penetrated the subsurface soils under the static weight of the drilling rods and/or hammer apparatus, and dynamic impact from the hammer was not required to drive the sampler. These sample locations are depicted on the boring logs by an SPT blow count designation of WR (weight of rods) or WH (weight of hammer).

In situ vane shear strength testing and laboratory testing of Shelby tube samples in this area also indicate the soils are highly compressible and exhibit low to very low shear strengths.

3.0 LABORATORY AND IN-SITU TESTING

Numerous relatively undisturbed samples (obtained from Shelby tubes) were tested in the laboratory to quantify geotechnical engineering parameters including shear strength and potential settlement/swell properties. One-dimensional consolidation tests and triaxial shear strength tests were performed to quantify these geotechnical properties. Geotechnical index testing including gradations, Atterberg limits, specific gravity, and moisture content were also performed on most of these samples as well as many of the Standard Penetration Test (SPT) split spoon samples. Moisture density relationships were also developed by the District as part of their soil survey. The table below summarizes the average results of index tests performed on samples obtained along the alignment.

Table 2 Average Properties of Predominant Soil Types.

AASHTO Soil Class.	USCS Soil Class.	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Liquidity Index	Minus #200 Sieve
A-4	SILT	24.2	NP, 26.2	NP, 19.4	NP, 6.8	0.7	91.6
A-6	Lean CLAY	28.7	34.8	21.1	16.9	0.6	92.1
A-7-6	Lean CLAY, Fat CLAY	36.6	47.0	24.9	22.1	0.6	96.5

Notes:
1. A-4 values based on results of 92 index tests. A-6 values based on results of 27 index tests. A-7-6 values based on results of 32 index tests.
2. 39 of the 92 Atterberg limit tests on the A-4 soils indicated the soils were nonplastic. The values given for the A-4 soils are the average results of the samples that were plastic.

The following observations can be made from Table 1:

1) The average water content of all three soil types was higher than the average plastic limit;
2) The average water content of the A-4 soils was just below the average liquid limit;
3) The average liquidity index for the soils ranged from 0.6 to 0.7; and,
4) The average percent of material finer than the number 200 sieve was between 92 and 97 percent.

All of these factors indicate that these soils are highly moisture sensitive and construction equipment is likely to have problems operating on these soils, especially during spring thaw and periods of wet weather. The purely mathematical “average” of the liquidity indices in the A-4 group is slightly misleading because a few samples contained very high values that significantly increased the average value. Neglecting these isolated cases provides a more realistic average of about 0.4 in the A-4 soils which is still indicative that the soils are moisture sensitive.

The following table indicates Station locations corresponding to Geotechnical Section borings and Soils Survey borings where liquidity indices (liquidity indices are calculated from the plasticity characteristics and in situ moisture content of the soil) 0.2 or higher were encountered within the top approximate 6 feet in fill areas or at depth in a cut area.

Boring Number	Approximate Station	Depth (ft)	Liquidity Index	In-situ Moisture (%)	Cut/Fill
D1-06	040+00	5.6	0.53	25	Fill
1289-1	049+80	15-16.5	0.27	36	Cut
HA-16	054+90	1.5-2.8	0.20	25	Fill
HA-16	054+90	3-4	0.32	26	Fill
1289-S18	61+00	7.5-9.0	0.28	30	Fill
D2-06	098+00	5.2	0.85	31	At grade
D3-88	128+00	5	1.29	27.6	At grade
D3-06	143+00	5.4	0.23	24	At grade
D4-88	236+00	6.0	0.95	27	Cut
1289-8	243+00	4.5-6.0	1.65	70	Fill
HA-10	243+50	0-1	0.83	81	Fill
1289-7	250+00	2.5-4.5	2.50	95	Fill
HA-9	282+00	12-24	0.20	28	Cut
1289-S8	300+14	5.0-6.5	1.02	31	Fill
1289-10	345+80	5.0-6.5	0.41	31	Cut
1289-11	367+35	7.5-9.5	3.09	107	Fill
D7-88	376+00	6	0.82	27	Cut
D9-06	389+00	6.1	0.52	23	Cut
HA-4	387+80	1.7-2.5	0.24	28	Fill
1289-12	398+00	5-7	1.05	52	Fill
1289-S4	416+00	5-6.5	0.52	28	Fill
1289-16	549+40	5.0-6.5	1.08	29	Fill

Results of the consolidation and laboratory shear strength testing indicate the in-situ silt and clay soils are highly compressible and exhibit relatively low shear strengths. The lowest shear strengths and most compressible soils are located from approximate Station 240+00 to 253+00 corresponding to the swampy area where organic soils are present.

Results of the in situ vane shear strength tests indicate a wide range of undrained shear strength depending upon the soil type, if organic material is present, depth, etc. In general, the shear strengths of the peat or silt soils containing organic material is very low and strengths generally increase where organic soils are not present.

For additional information regarding the subsurface conditions, see the attached boring logs in the Appendix.

4.0 DESIGN AND CONSTRUCTION RECOMMENDATIONS

As was briefly discussed within sections 2.0 and 3.0 of this report, moisture sensitive silts and clays are present throughout the project and in some areas are soft to very soft. Embankment foundation treatments will be required to construct the proposed embankment over these soft ground conditions. In some areas we anticipate that larger equipment typical of that generally used in road construction (such as scrapers or larger bulldozers) will not be supported by the soft ground and excessive pumping or bearing capacity failures under the wheel loads are likely.

At approximate Stations 240+00 to 253+00 the soft soils contain organic material and extend to depths greater than 10 feet and continuously in a lateral direction between our borings. Groundwater was generally encountered within 5 feet of the surface or higher, depending upon the time of investigation. Therefore, simply removing these soils and replacing them with more functional soil is not feasible and performing ground improvement over this large of an area will be cost prohibitive. In addition to the construction difficulties that might be expected, long-term settlement and slope instability will also need to be mitigated within this area. We believe the proposed embankments can be successfully constructed in this location; however, controlled rates of fill placement (i.e. staged construction with instrumentation monitoring), the use of high strength geosynthetics and special borrow, and the use of light weight equipment will be required. The special borrow used for the pavement section, depending upon the actual gradation, might also be suitable for construction of the embankments in this area. It will also be imperative that the special provisions for construction in this area are followed by the Contractor and strictly enforced by the construction project manager. The potential of possibly moving the alignment out of the swampy area should also be evaluated. Although this would require additional design time there could be significant cost savings and we recommend that this be at least briefly evaluated.

These soft areas and design and construction recommendations will be further discussed throughout this report. We also recommend that the MDT project manager assigned to this project receive a copy of this report prior to construction for their review to help them understand the significance of the geotechnical conditions to be expected on this project. We further anticipate that representatives from the Geotechnical Section will need to be on-site during critical aspects of construction to read geotechnical instrumentation, and provide guidance to the Project Manager with respect to geotechnical conditions.

4.1 Embankment Foundations

Proposed embankment foundation areas were evaluated for support characteristics and constructability throughout the project. Geotechnical recommendations are based on observations during the field investigation, subsurface investigation, laboratory testing results, soil survey results obtained by the district, and previous experience with similar soils.

There are several different areas where the proposed alignment will traverse through areas delineated as wetlands or other areas containing numerous cattails, vegetation, or phreatophytes indicative of subgrade soils with high moisture contents, high organic contents, or a relatively high water table. Our field investigation encountered organic soils (peat) or soils containing high percentages of organic material in several different locations.

Liquidity indices (determined by the in situ moisture content and plasticity characteristics of the soil) of near surface soils ranged from negative values to approximately 3.1. At values above approximately 0.3 the soil can become difficult to handle, place, and compact and at values above 0.5 to 0.6 the soil is very difficult to handle, place, or compact.

There are other select locations where liquidity indices were greater than 0.2, not indicated in the table presented in the laboratory testing section of this report, however these locations were generally in excess of 3 meters in depth in a proposed embankment fill area. It should be noted that in some locations, Atterberg limit tests were not performed on the samples nearest the ground surface and liquidity indices could be higher than those presented in this report.

Based upon this information and the information presented in the subsurface soils section, we recommend embankment foundation treatment consisting of placement of stabilization geotextile, biaxial geogrid, and special borrow at the locations in the table below (Note: this table does not include the area from Stations 240+00 to 253+00, as this area will be discussed separately). These recommendations are primarily predicated on constructability of the embankments over the soft soils.

Station Range		**Side**
34+50	38+10	Lt, Rt
53+50	56+00	Lt, Rt
60+00	65+00	Lt, Rt
64+00	68+00	Lt
155+00	156+00	Lt.
212+30	213+20	Rt.
366+50	368+00	Lt, Rt
396+50	402+50	Lt, Rt

The use of geogrid could possibly be eliminated in some of these locations dependent upon the water levels at the time of construction and size of construction equipment used. Since we cannot predict the size of equipment that will be used or the water elevations at the time of construction, we recommend assuming geogrid will be required and including this quantity in the contract plans for bidding purposes. The potential for selective culling of geogrid in these areas will likely need to be assessed by the project manager once construction begins and the presence (or lack thereof) of excessive pumping of the subgrade soils below wheel loads, can be determined.

We do not anticipate embankment foundation treatment at other locations (other than Stations 240+00 to 253+00) However, if significant precipitation occurs, or earthwork is attempted by the contractor during spring thaw or during precipitation events or shortly thereafter before the near surface clay and silt soils have had sufficient time to dry, sub excavations may be required along with placement of stabilization geotextile, geogrid, and special borrow to stabilize the foundation soils. Localized foundation treatment may also be in low-lying areas created during construction that collect run off water.

<u>Station 240+00 to 253+00</u>

Subsurface soils from approximate Stations 240+00 to 253+00 are organic and very soft (see boring log numbers 1289-08, 1289-S11, 1289-07, and 1289-S10). Due to the subsurface conditions within this area, typical embankment construction using scrapers and large dozers will not be possible as the ground will not support this size of equipment. Large/heavy equipment will cause bearing capacity failures of the near surface soils and placing embankment fill too quickly could also cause larger scale global failures of the embankment. To mitigate this, we recommend constructing the embankments in this area with a high strength geogrid and stabilization geotextile placed on the existing ground surface, using a free draining granular soil for the entire embankment fill, and placing the fill with lightweight equipment. The embankment should not be used as a haul route and scrapers should not be allowed to haul across the embankment until the subsurface soils have had adequate time to consolidate and gain strength.

Our analyses indicate total settlements of approximately 4 feet are likely under the proposed fill heights of approximately 5 to 7 feet. Based upon this predicted settlement, we are also recommending to surcharge this area after the embankment is constructed to grade and pore water pressures have dissipated to accelerate the time required for settlement. That is, the embankment will need to be constructed to the design subgrade elevation, construction stopped to allow excess pore pressures to dissipate, and then the surcharge fill placed. It would be very beneficial if the project letting and construction timing occur to allow the surcharge to be left in place over the winter months when construction is stopped. Three feet of the predicted settlement is expected to occur during construction, assuming that the foundation soils below the embankment are allowed to settle under the surcharge load during the winter and the roadway is paved during a second construction season. Approximately 1 foot of the settlement is expected to occur due to secondary consolidation in the organic soil and will continue after construction is completed.

The construction methods and monitoring required through this area will also require the installation and monitoring of geotechnical instrumentation to evaluate pore water pressures and settlement to ensure that pore water pressures are not excessive during construction and to monitor settlement and determine when the surcharge load can be removed. We will provide a special provision to address construction through this area after we receive any comments or concerns from the District, Construction or other design units with this proposed approach.

Settlement below embankments

Proposed embankment foundation areas were also evaluated for total settlement. Based on our analyses of representative embankments, foundation settlement estimates range from a few inches up to approximately 4 feet in the area between Stations 240+00 to 253+00. The majority of the predicted settlements are on the order of 3 to 8 inches. The Table below provides select locations, proposed fill heights and corresponding predicted settlements. The settlement estimates are directly proportional to the proposed fill height, the thickness of the compressible subsurface layer(s), and the anticipated water elevation.

Location	Approximate fill height, ft.	Estimated settlement, in.
63+00-65+00	13	20
115+00	8	5
155+80	16	11
179+80	12	13
240+00 – 253+00	6-7	48
300+00	10	4
367+40	6	17
442+40	13	12
509+00	13	5
549+40	17	17

The proposed alignment typically utilizes the existing PTW embankment within the new embankment prism. This greatly increases the chances for differential settlement in the constructed embankments. The subsurface soils below the portion of the newly constructed embankment that contains the PTW have already settled but the widened area where the PTW is not present will settle under the applied loading. This will cause differential settlement in the area where the new embankment abuts against the existing embankment.

Due to the size of this project, we recommend that some of the settlement be allowed to occur during the winter before the roadway is paved during the following construction season. We believe paving during a second construction season is feasible due to the size of this project and anticipate that completion of the contract will require a minimum of two construction seasons. This will greatly help mitigate the detrimental effects of settlement by allowing it to occur both during construction and during the winter before the roadway is paved. In locations where organic soils are present, some additional settlement should still be expected.

The approach embankments at the bridge location should be constructed before driving piles. This will allow most of the settlement within the subsurface soils induced from embankment construction to occur and reduce the magnitude of negative skin friction along the piles. The approach embankments should be constructed and foundation soils allowed to settle for a minimum of 30 days (or longer if possible) before piles are driven. Negative skin friction causes additional downdrag loading on the piles thus larger piles, higher ultimate capacities during driving, and potentially longer piles are required when negative skin friction is not mitigated. Reducing the potential amount of negative skin friction by constructing the approach embankments prior to driving piles is recommended. Further recommendations related to the bridge foundations and approach embankments will be provided in our Geotechnical Activity 466 - Structures report.

4.2 Embankment Slope stability

Depending on the height of the fill and requirements to avoid wetlands, the majority of the proposed fill slopes for this project are either 6:1, 4:1 or 3:1. Preliminary slope ratios at the bridge approaches are shown at 2:1.

The proposed embankments at representative locations have been analyzed (where applicable) for both the end of construction case (using undrained shear strengths) and long-term case (using drained shear strengths) slope stability. The end of construction case is applicable to fine-grained soils (such as silts or clays) where the in situ drainage characteristics of the soil coupled with the relatively fast rate of loading (fill placement) from construction will cause excess pore water pressures to develop. These increases in pore water pressure decrease the strength of the fine grained soil. That is, the fine grained soils are weakest immediately after the embankment is constructed when the pore water pressures are presumably at their highest, and the strength of the subsurface soil slowly increases with time as the excess pore water pressures are dissipated. Once the excess pore pressures have dissipated, the fine grained soils are analyzed with drained shear strengths. Undrained conditions are not applicable to coarse-grained sands and gravels, as the increases in pore water pressures are expected to dissipate rapidly during construction and these soils are therefore analyzed with drained strengths only.

Short term stability

The proposed embankments are unstable under undrained loading conditions from Stations 240+00 to 253+00 for fill heights greater than approximately 6 feet (potential surcharge heights). These embankments are unstable at the proposed 3H:1V due to the soft organic subsurface soils, high ground water, and proposed embankment height.

Several options are available for constructing the embankments through this section including performing at depth ground improvement, staged embankment construction, and constructing stabilizing toe berms or flattening slopes, amongst others. We believe performing ground improvement will be highly cost prohibitive due to the large area that would be required and that the other two options should be evaluated. Constructing embankments to the proposed grade utilizing staged construction to allow excess pore water pressures to dissipate and corresponding shear strengths to increase before the embankments are constructed to their full design height is in our opinion the most viable option. Alternatively, stabilizing toe berms or flatter slopes on both sides of the embankment could also be constructed, however this will impact additional wetlands. We request input from the District on the preferred option.

Embankments that are constructed in stages will require installation of piezometers, settlement plates, and possibly inclinometers before fill placement begins and the instrumentation will need to be monitored during construction. The piezometers will enable monitoring of pore water pressures during construction to determine when fill placement must be stopped to allow excess pore water pressures to dissipate, and when fill placement can be resumed after the pore water pressures have had sufficient time to dissipate. Settlement plates will be monitored to determine when a sufficient amount of settlement has occurred and when paving can occur. Two options are available for this including; the Geotechnical Section (or designated representative) installing instrumentation prior to letting of the contract, or including the work within the contract documents and the contractor could retain a sub consultant to install the instrumentation. Regardless of what option may be selected, the Geotechnical Section would evaluate the instrumentation data and this would not be performed by the Contractor.

The proposed embankments at the bridge approaches (Approximate Stations 63+00 to 65+00) are marginally stable with the proposed 2:1 slopes under undrained loading condition. Similar to the embankments from Station 240+00 to 253+00 the stability is controlled by the poor foundation soils present. Our estimated factors of safety against slope instability are slightly below FHWA recommended minimum criteria. Improvements to the slope stability factors of safety can be achieved using the same methods at previously discussed for the area from 240+00 to 253+00 (flatter slopes or staged construction).

Long Term Stability

Fill slopes at Sta. 240+00 to 253+00 that will be constructed over very soft organic peat and silt deposits were analyzed for long term slope stability. The proposed 3H:1V embankment slopes are stable for long term conditions due to the relatively low fill heights but will require staged construction as indicated previously to mitigate short term conditions and allow placement of the surcharge.

We anticipate the material excavated from cut sections that will be used for embankment construction in other areas of the project will contain a significant portion of A-4, A-6 or A-7 soils. It is imperative that moisture and compaction specifications are adhered to during construction to ensure stability of the fill slopes that utilize these soils.

This project is to be constructed in lacustrine clays, silts and fine sand soil types that are highly susceptible to erosion and hard to re-establish vegetation. We recommend an aggressive slope erosion protection and revegetation plan.

4.3 Cut Sections

Proposed cuts are all 2H:1V or flatter with the majority of the slopes at 3H:1V or flatter. We anticipate these proposed cut slopes will be stable in their proposed slope ratios. As with the embankments, this project is to be constructed in lacustrine clays, silts and fine sand soil types that are highly susceptible to erosion and hard to re-establish vegetation. We recommend an aggressive slope erosion protection and revegetation plan.

Through cuts are anticipated from approximate Stations 45+50 to 52+50 and from 79+00 to 87+00. Due to the moisture sensitive soils present, excavation and handling these soils could be difficult in this area if the excavation is attempted during the wet season or shortly after precipitation events.

A rock outcrop is present just above the existing ditch section in the vicinity of Station 496+00. We recommend that the district survey this rock outcrop for exact location from the proposed alignment to determine if the small back slope cuts or construction of the new ditch section within this area will encounter the rock outcrop. In the event the survey indicates the outcrop will be encountered we recommend that the potential to raise the grade or shift the alignment left be investigated to avoid the rock. We anticipate the rock outcrop will require some limited blasting if encountered. We should be notified after the survey data is collected and if avoidance is not possible and the Geotechnical Section can provide further recommendations, as necessary for excavation/blasting of the rock outcrop.

4.4 Culverts

Geotechnical investigations were performed for the majority of proposed culverts larger than 36 inches in diameter. At those locations where culverts 36 inches or smaller are anticipated, geotechnical investigations were generally not performed. The table below depicts estimated settlement near the center of the culvert at the proposed culvert and/or stockpass locations assuming that foundation treatment (if required) is performed in accordance with the attached special provision.

Approximate Station	Proposed Culvert/Stockpass Diameter	Approximate Settlement Inches (in)	Foundation treatment required
*115+00	95"	2	Yes
123+00	36"	4(check)	No
156+30	48"	8	Yes
180+00	48"	4	Yes
*277+40	120"	Not investigated	
*295+29	120"	4	yes
*300+14	120"	2	yes
367+25	13'-6" x 9'-6"	17	Yes
*442+40	95"	7	Yes
*508+75	95"	4	Yes
*549+76	95"	14	No

* Indicates a Stockpass

Culvert foundation treatment consisting of replacing a portion of the subsurface soils at and below the bearing elevation with imported granular material improves constructability and indirectly reduces some of the settlement. The estimates in the table within this section of the report assume that foundation treatment will be performed where recommended.

Based upon the information presented above, we recommend culvert foundation treatment consisting of placement of stabilization geotextile and foundation material at all of the culverts in the table above to improve constructability and reduce a portion of the settlement by reducing the thickness of the compressible soils. Dewatering will also be required at some of the culverts.

The predicted settlements at Stations 367+25 and 549+76 are excessive. Several options are available to mitigate some of the settlement at these structures. A surcharge could be constructed with a temporary culvert installed and the settlement would then be allowed to take place, at which point the surcharge and temporary culvert could be removed and the permanent culvert installed and embankment constructed. Alternatively, oversized structures that will be hydraulically sufficient after settling and silting in could be installed, but the predicted settlement of 17 inches may still be excessive and cause damage to the culvert. Larger structures would also be easier to sleeve or repair when damaged by differential settlement. Other options such as performing ground improvement below the structure footprint could also be performed, however we anticipate the mobilization and cost for such a small project will be quite expensive. A final option would be to consider a bridge at this location. The Geotechnical Section requests input from Hydraulics, Road Design, CES Bureau and the District as to what option is preferable. Upon receiving this input, additional recommendations and a special provision to also include dewatering will be provided.

4.5 Moisture Sensitive Soils

As discussed previously within this report, moisture sensitive soils are located throughout the project in both cut and fill areas. Small increases in moisture content are detrimental to the shear strength of these soils and could result in construction difficulties. In-situ moisture contents (at the time of our investigation) at many locations are already high and will require adequate processing to allow the soils to dry before they can be effectively handled or placed.

The contractor should be made aware of this, and should plan construction sequencing to minimize the volume of soil without vegetative cover exposed to precipitation events, and should anticipate stopping earth work during precipitation events and not resuming earthwork until sufficient drying of the soils has occurred. A Special Provision for moisture sensitive soils is included as an attachment to this report.

4.6 Grading Material (Shrink/Swell)

Project unclassified excavation soils classify within the A-4, A-6, or A-7 groups in accordance with AASHTO. These same soils classify as silts or clays in accordance with the Unified Soil Classification System (USCS). Based on the soils encountered and the in-situ relative densities or consistencies, we estimate the in-situ soils will undergo a 25 to 35 percent volume reduction when compacted to 95 percent of the maximum dry density in accordance with MT 210. This volume reduction estimate includes the compaction of the in-situ soil, and loss of material due to grading and haul operations. Since the predominate soil type located on this project consists of fine grained silt or clay and due to numerous variables and uncertainty in evaluating shrink/swell information, we do not believe any type of improvement to the mass-haul diagram or earthwork balance can be gained by attempting to break out the shrinkage estimates by station range, especially since the individual values will all be near the previously recommended values. We did not encounter any materials during our investigation that would be expected to undergo a volume expansion (swell) when excavated and compacted other than the rock outcrop previously discussed. This very limited volume of material in the rock outcrop near Station 496+00 would be expected to swell 10 to 15 percent.

4.7 Seismicity

The site lies within the Intermountain Seismic Belt and is mapped as a zone of potentially significant seismic ground movement. Based on the fine grained nature of the soils, we anticipate that the risk of liquefaction or of lateral spreading is low in most areas. Earthquake induced settlement and slope movement is possible due to shear strength reduction in the fine grained soils due to dynamic loading. Based upon the required costs associated with constructing cut slopes or embankments to withstand seismic loading, we anticipate the Department will not elect to mitigate the potential for seismically induced slope instability or settlement and should damage occur from an earthquake, the damage will be repaired at that time. We should be notified if this assumption is incorrect or if additional recommendations with respect to seismic stability are requested. We will provided further recommendations for the bridge area as part of our 466 report.

4.8 Special Borrow/Typical Section

The proposed typical section consists of placing 2 feet of special borrow (A-1-a with a minimum R-value of 60) below the base course. Since nearly all soils located within the project limits are fine grained, a significant concern is the potential for the fine grained soils to infiltrate the base course. Gradation testing of base course samples obtained below the existing pavement section resulted in fines contents ranging from approximately 5 to 16 percent, indicating some fines may have infiltrated into the base course over time assuming that the base course did not originally contain up to 16 percent fine grained material.

Infiltration of the base course with the fine grained soils will reduce the pavement life and may require more intensive maintenance. It is our opinion that the poor condition of the existing roadway can be at least partly attributed to the migration of the fines into the base course. The use of a separation or stabilization geotextile below the base course would minimize the migration of the fines into the base course and improve the pavement longevity.

We understand the use of geotextile below the special borrow is already planned in cut areas throughout the project and agree with this design concept. Based upon the existing pavement condition, we recommend that the geotextile also be included (at a minimum) from approximate Stations 67+00 to 253+00, where FWD data indicates very low resilient moduli. Although installing a geotextile will increase project costs, these associated costs should be evaluated against the life cycle costs of replacing the pavement section prematurely. Since a geotextile will be placed in the cuts and we have recommended placing geotextile from Station 67+00 to 253+00, the required quantities will be relatively high which should theoretically reduce unit costs and it may be desirable to install the geotextile throughout the project limits. The use of a geotextile in the fill areas is not as critical from 253+00 to the end of the project, but should still be evaluated.

5.0 Limitations

Professional judgments and recommendations are presented in this report. They are based partly on evaluation of the technical information gathered, partly on historical reports and partly on the Geotechnical Section's general experience with Glacial Lake Missoula sediment subsurface conditions in the Missoula District. The Geotechnical Section does not guarantee the performance of the project in any respect other than that the engineering work and the judgment rendered meet the standards and care of the profession. It should be noted that the borings may not represent potentially unfavorable subsurface conditions between borings. If, during construction, soil, rock, or water conditions are encountered that vary from those discussed in this report or historical reports, or if alignment and grade changes are required, the Geotechnical Section should be notified immediately in order that it may evaluate effects, if any, on our recommendations. The recommendations presented in this report are applicable only to this specific site. These data are not to be used for other purposes.

Due to the presence of the poor soils located on this project a meeting may be required to discuss the various geotechnical recommendations and options provided herein, this could als be discussed at the upcoming Plan-In-Hand meeting. If there are any questions regarding the report or a meeting is requested, please contact Jeff Jackson by phone at (406) 444-3371 or email at jejackson@mt.gov or Bret Boundy at (406) 444-6278 or via email at bboundy@mt.gov.

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Geotechnical Correspondence File

Attachments:
- Draft Boring Logs
- Boring Log Key
- Materials Form 111
- Summary of Hand Auger and Grab Samples
- Summary of Soil Index Tests